Ovidius University Annals Series: Civil Engineering, Year 26, 2024
DOI: 10.2478/ouacsce-2024-0007

Comparative study between analytical calculation according to SR-EN 1993-1-8 and calculation of a joint using the Idea Statica software

Florin Țepeș-Onea[1], Andreea Constantinescu[2*], Andrei-Cristian Panchici[3]

1 "Ovidius" University Constanta (e-mail: tflorin@univ-ovidius.ro)
2* "Ovidius" University Constanta (corresponding author, e-mail: constantinescu_10@yahoo.com)
3 Technical University Civil Engineering of Bucharest (e-mail: andreipanchici@gmail.com)

Abstract **– The study proposes a comparison between the analytical verifications provided in Eurocode 3: Design of steel structures Part 1-8: Design of joints for a beam-column joint with reduced end plate and the same verifications performed by the IDEA StatiCa software. The starting point was a metal structure project with a height regime of P+1, with a span of 6 meters, an opening of 6 meters and a level height of 3.70 meters, designed for the area of Constanta. The steel structure was designed with one direction centrically braced and the other unbraced. The structure has 13 cm thick composite concrete floors and the terrace on the top level is non-accessible. The joint analyzed in this paper is located in the braced direction.**

Keywords – reduced end-plate joint.

1. INTRODUCTION

Our goal is to optimize the joint in particular by using the automatic calculation program IDEA StatiCa, which is based on the finite element method, as a much simpler solution than an analytical calculation and to obtain some conclusions that can be used in the design process.

For a metal building with a height regime P+1, we propose to calculate one of the articulated joints, both with the SR-EN 1993-1-8, and with the help of the IDEA StatiCa software, in order to compare the results obtained, in order to ensure the most comprehensive design each time.

In this regard, we started from the calculation of the P+1 steel structure and its optimization so that both the specific provisions of the P100-1/2013 standard, braced and not, and the resistance and stability check for the structural elements are respected. [1] Subsequently, the calculation was made using the 2 methods

The chosen metal structure is an office building, with a height of GF+1, with a span of 6.00 meters, an opening of 6.00 meters and a level height of 3.70 meters.

The structure was designed for the Constanta area, with the peak ground acceleration for design, ag, being equal to 0.2g. The structure is not sensitive to the vertical component of seismic action. Only the two horizontal components described by the elastic response spectra for accelerations Se(T)=ag $\cdot$ β(T) are taken into account. It is designed according to the DCM ductility class.

ISSN 2392-6139 / ISSN-L 1584-599

The building was designed in a centrically braced direction and the other direction unbraced. Columns with HEB450 section and beams with IPN400 section were used. The structure of the resistance structure is made of laminated profile made of S235 steel laminated profile. The structure has composite reinforced concrete floors, with a thickness of 13 cm, and the terrace on the top level is non-circular. The joint that is the subject of this work is positioned on the unbraced direction on axis 2 where the column-beam joints will be articulated. Along axes 1 and 3 on the unbraced direction, the joints will be rigid.

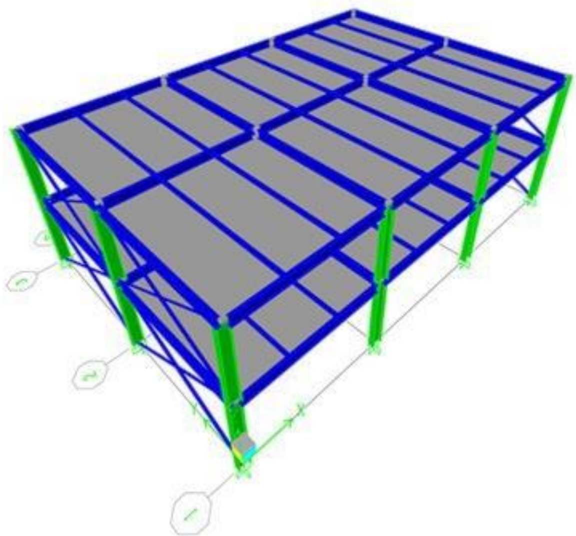


Fig.1 The metal structure studied

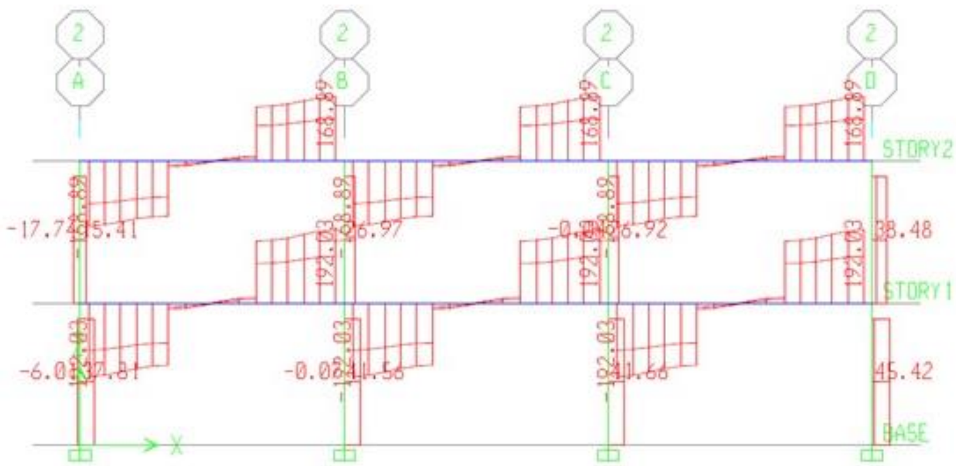


Fig. 2 Shear Force Diagram Results

The metal structure was calculated using the ETABS software, which operates with the finite element method. The shear force diagrams resulting from the ultimate limit state were determined, as the beams are dissipative elements. The maximum calculated shear force was 192,03 kN.

2. REDUCED END PLATE JOINT

A joint is represented by its physical components that connect the column and the beam, and it is located where the connection is made. It consists of the components that form the joint, being characteristic of this typology (for example, in the case of the joint with an end-plate with bolts, the bolts and the end-plate are involved). [3]

The end plate connection with bolts can have various characteristics depending on the variation of internal parameters: bolt diameter, end plate thickness, presence of stiffeners, component resistance, etc. [3]

The plate will be attached in the factory by welding the end of the beam. The welding will be a fillet weld. On-site, the assembly joint is made with one or more double rows of bolts arranged vertically. The height of the plate will not exceed the height of the beam. This is an inexpensive solution, easy to manufacture in the factory but more difficult to assemble due to small tolerances between the distance between columns and the beam dimensions. [3]

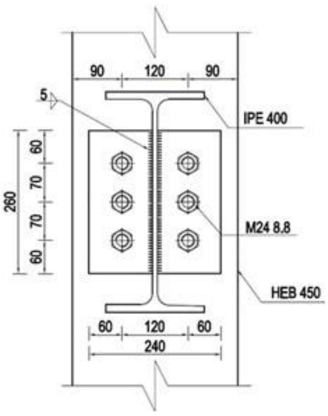


Fig. 3 Calculated Joint

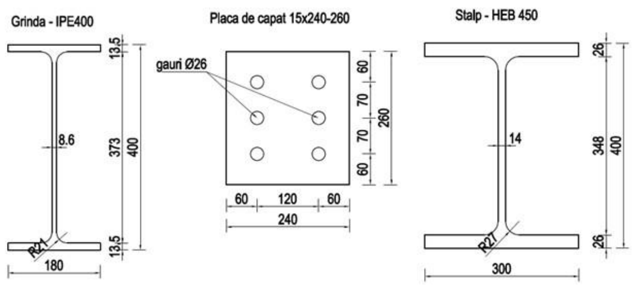


Fig. 4 Components of the Verified Joint

The main components of the joint:

Configuration: connection between the beam end and the column flange
Column: HEB 450
Beam: IPE 400
Joint Type: End plate connection
End plate: Pl 260 x 240 x 15
Bolts M24 g 8.8

Calculation of joints according to SR EN 1993-1-8 [4], [5], [6]
Rotation Requirements

$$h_p \leq d_b \tag{1}$$

$$d_b = h - 2t_{bf} - 2r = 400 - 2 \cdot 13{,}5 - 2 \cdot 21 = 331mm > 260mm \quad Verified$$

where:
h – the height of the beam section;
t_{bf} – thickness of the base of the beam;
r – beam bend radius.

Ductility requirements

$$\frac{d}{t_p} \geq 2{,}8 \cdot \sqrt{\frac{f_{yp}}{f_{ub}}} \tag{2}$$

where:
d – diameter of the bolt rod;
f_{yb} – yield stress;
f_{ub} – ultimate tensile strength.

$$\frac{d}{t_p} = \frac{24}{15} = 1{,}6 \tag{3}$$

$$2{,}8 \cdot \sqrt{\frac{f_{yp}}{f_{ub}}} = 2{,}8 \cdot \sqrt{\frac{235}{800}} = 1{,}51 < 1{,}60 \rightarrow \text{ Verified} \tag{4}$$

$$0{,}4 \cdot t_{wb} \cdot \beta_w \cdot \sqrt{3} \cdot \frac{f_{ybw}}{f_{ubw}} \cdot \frac{\gamma_{M2}}{\gamma_{M0}} \tag{5}$$

where:
βw – correlation coefficient for the evaluation of weld strength;
t_{wb} – the thickness of the web of the beam;
f_{ybw} – yield stress;
f_{ubw} – ultimate tensile strength;
γ_{M2}, γ_{M0} – partial safety factors.

$$0{,}4 \cdot 8{,}6 \cdot 0{,}8 \cdot \sqrt{3} \cdot \frac{235}{360} \cdot \frac{1{,}25}{1{,}00} = 3{,}89 < 5 \rightarrow Verified \tag{6}$$

Shear node resistance
Bolt rod shears

$$V_{Rd1} = 0{,}8 \cdot n \cdot F_{\vartheta Rd} = 0{,}8 \cdot 6 \cdot 137{,}5 = 660\ kN \tag{7}$$

$n = 6$ $- number\ of\ bolts$

Shear resistance of a bolt $F_{\vartheta Rd}$

$$F_{\vartheta Rd} = \alpha_\vartheta \cdot A \cdot \frac{f_{ub}}{\gamma_{M2}} = 0{,}6 \cdot 353 \cdot \frac{800}{1{,}25} = 137{,}5\ kN \tag{8}$$

where:
αθ – reduction factor;
A – tensile stress area of the bolt;
f_{ub} – ultimate tensile strength of the bolt.

$$F_{b,Rd} = \frac{k_1 \cdot \alpha_b \cdot d \cdot t_p \cdot f_{up}}{\gamma_{M2}} = \frac{2{,}5 \cdot 0{,}65 \cdot 24 \cdot 15 \cdot 360}{1{,}25} = 168{,}48\ kN \tag{9}$$

$$\alpha_b = \min(\frac{e_1}{3 \cdot d_0}; \frac{p_1}{3 \cdot d_0} - \frac{1}{4}; \frac{f_{ub}}{f_{up}}; 1,0) = \min(0,77; 0,65; 2,20; 1,00) = 0,65 \quad (10)$$

where:
e_1 – the longitudinal distance from the edge;
d_0 – hole diameter;
p_1 – Distance between bolts in the longitudinal direction.

$$k_1 = \min(2,8 \cdot \frac{e_2}{d_0} - 1,7 ; 2,5) = \min(4,76; 2,5) = 2,5 \quad (11)$$

$$V_{Rd,2} = n \cdot F_{b,Rd} = 6 \cdot 168,48 = 1010,88\ kN \quad (12)$$

Pressure on the hole in column flange
Pressure resistance on the hole of a bolt

$$F_{b,Rd} = \frac{k_1 \cdot \alpha_b \cdot d \cdot t_{cf} \cdot f_{ucf}}{\gamma_{M2}} = \frac{2,5 \cdot 0,65 \cdot 24 \cdot 21 \cdot 360}{1,25} = 235,87\ kN \quad (13)$$

Where:
t_{cf} – column flange thickness.

$$k_1 = \min(2,8 \cdot \frac{e_{c2}}{d_0} - 1,7 ; 2,5) = \min(6,9; 2,5) = 2,5 \quad (14)$$

Where:
e_{C2} – the distance from the last row of bolts to the edge

$$V_{Rd,3} = n \cdot F_{b,Rd} = 6 \cdot 235,87 = 1415,23\ kN \quad (15)$$

Rough Section End Plate Shear

$$V_{Rd,4} = \frac{2 \cdot h_p \cdot t_p}{1,27} \cdot \frac{f_{yp}}{\sqrt{3} \cdot \gamma_{M0}} = \frac{2 \cdot 260 \cdot 15}{1,27} \cdot \frac{235}{\sqrt{3} \cdot 1,0} = 833,32\ kN \quad (16)$$

Cutting the end plate in the net section

$$V_{Rd,5} = 2 \cdot A_{\vartheta net} \frac{f_{up}}{\sqrt{3} \cdot \gamma_{M2}} = 2 \cdot 2730 \frac{360}{\sqrt{3} \cdot 1,25} = 907,90\ kN \quad (17)$$

where:
$A_{\theta net}$ – net cross-sectional area

$$A_{\vartheta net} = t_p \cdot (h_p - n_1 \cdot d_0) = 15 \cdot (260 - 3 \cdot 26) = 2730\ mm \quad (18)$$

where:
n_1 – number of bolt rows

Beam core failing

$$V_{Rd,6} = A_{\vartheta} \cdot \frac{f_{yb1}}{\sqrt{3} \cdot \gamma_{M0}} = 2012,4 \cdot \frac{235}{\sqrt{3} \cdot 1,0} = 273,05\ kN \quad (19)$$

where:
A_ϑ – tensile stress area of the bolt

$$A_\vartheta = 0{,}9 \cdot h_p \cdot t_{wb1} = 0{,}9 \cdot 260 \cdot 8{,}6 = 2012{,}4\,mm \tag{20}$$

For shear fail, the minimum value is:

$$V_{Rd,6} = 273{,}05\,kN$$

Failure Mode: Beam core failing
Shear force at end: 192.03 kN
Shear force: 273.05 kN
Verified: $273{,}05 > 192{,}03$
Elongation of bolts

$$N_{Rd,u1} = n \cdot F_{tRd,u} = 6 \cdot 231{,}05 = 1386{,}3\,kN \tag{21}$$

$$F_{tRd,u} = \frac{k_2 \cdot f_{ub} \cdot A_s}{\gamma_{M4}} = \frac{0{,}9 \cdot 800 \cdot 353}{1{,}1} = 231{,}05\,kN \tag{22}$$

where:
n – number of bolts;
$F_{tRd,u}$ – the tensile strength of a bolt;
k_2 – coefficient;
γ_{M4} – safety factor

Bending the end plate

$$N_{Rd,u2} = \min\,(F_{Rd,u,ep1}; F_{Rd,u,ep2}) \tag{23}$$

For node 1:

$$F_{Rd,u,ep1} = F_{T,1,Rd} = \frac{(8 \cdot n_p - 2 \cdot e_w) \cdot M_{pl,1,Rd,u}}{2 \cdot m_p \cdot n_p - e_w \cdot (m_p + n_p)} = \frac{(8 \cdot 60 - 2 \cdot 11{,}25) \cdot 4{,}76 \cdot 10^6}{2 \cdot 50{,}05 \cdot 60 - 11{,}25 \cdot (50{,}05 + 60)} = 459{,}62\text{ kN} \tag{24}$$

$$n_p = \min\left(e_2; e_{2,c}; 1{,}25 \cdot m_p\right) = \min(60; 80; 62{,}65) = 60\,mm \tag{25}$$

$$m_p = \frac{p_2 - t_{w1b1} - 2 \cdot 0{,}8 \cdot a\sqrt{2}}{2} = \frac{120 - 8{,}6 - 2 \cdot 0{,}8 \cdot 5\sqrt{2}}{2} = 50{,}05\,mm \tag{26}$$

$$e_w = \frac{d_w}{4} = \frac{45}{4} = 11{,}25\,mm \tag{27}$$

$$M_{pl,1,Rd,u} = M_{pl,2,Rd,u} = \frac{1}{4} \cdot \frac{h_p \cdot t_p \cdot f_{up}}{\gamma_{M0}} = \frac{1}{4} \cdot \frac{260 \cdot 15^2 \cdot 360}{1{,}1} = 4{,}79\,kNm \tag{28}$$

For Node 2

$$F_{Rd,u,ep2} = F_{T,2,Rd} = \frac{M_{pl,2,Rd,u} + n_p \cdot \sum F_{t,Rdu}}{m_p + n_p} = \frac{2 \cdot 4{,}76 \cdot 10^6 + 60 \cdot 1386{,}3 \cdot 10^3}{60 + 50{,}05} = 842{,}73\,kN \tag{29}$$

$$N_{Rd,u2} = \min\ (F_{Rd,u,ep1}; F_{Rd,u,ep2}) = 459{,}62\ kN \tag{30}$$

Elongation of the beam core

$$N_{Rd,u3} = \frac{t_w \cdot h_p \cdot f_{u,bw}}{\gamma_{M,u}} = \frac{8{,}6 \cdot 260 \cdot 360}{1{,}1} = 731{,}78\ kN \tag{31}$$

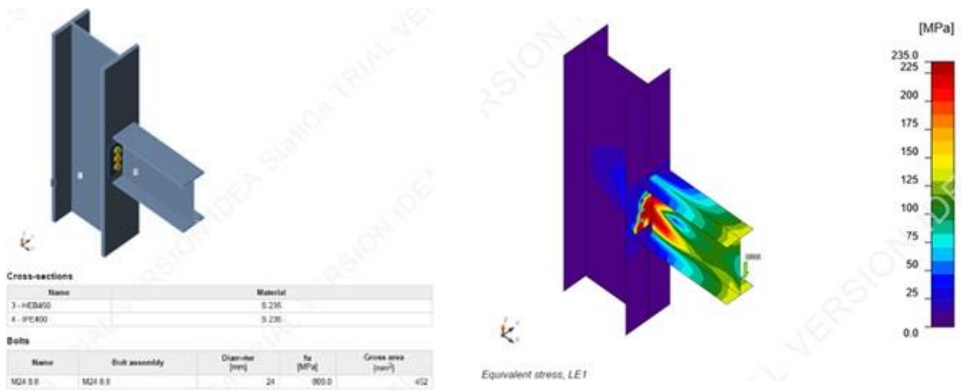


Fig. 5 The Joint Introduced in the IDEA StatiCa Program; Stress Distribution in the Joint

Resistance of node to tension
Failure mode: tension in the web of the beam

Nu=459.62 kN > Vsd=138.04 kN Control

where:
Vsd – design shear capacity.

3. CALCULATING THE JOINT WITH THE IDEA STATICA PROGRAM

The IDEA StatiCa program was used to verify the joint. Its main feature is that the modeling of joints is not based on predefined forms but follows the typology of the manufacturing process. [7]

Main features of the program:

- Stiffness analysis for any type of joint;
- Finite element model is generated automatically, without needing user creation;
- The node is modeled according to the manufacturing process—holes, bolts, cuts, stiffeners, welds, plates, etc.
- Calculation of forces and stresses in nodes based on finite element analysis in the elastic-plastic domain;
- Local buckling analysis and critical load factor analysis. [7]

As the joint has been checked at the design stress resulting from the ultimate limit state, the shear force load has been gradually increased to determine both the failure mode and the shear force value at which the joint will break.

Summary

Name	Value	Status
Analysis	100.0%	OK
Plates	1.3 < 5%	OK
Bolts	65.3 < 100%	OK
Welds	0.0 < 100%	OK
Buckling	Not calculated	

Plates

Name	Thickness [mm]	Loads	σ_{Ed} [MPa]	ε_{Pl} [%]	Status
C-bfl 1	26.0	LE1	187.4	0.0	OK
C-tfl 1	26.0	LE1	3.5	0.0	OK
C-w 1	14.0	LE1	93.6	0.0	OK
B-bfl 1	13.5	LE1	142.2	0.0	OK
B-tfl 1	13.5	LE1	142.1	0.0	OK
B-w 1	8.6	LE1	236.6	0.0	OK
EP1	15.0	LE1	237.8	1.3	OK

Design data

Material	f_y [MPa]	ε_{lim} [1e-4]
S 235	235.0	500.0

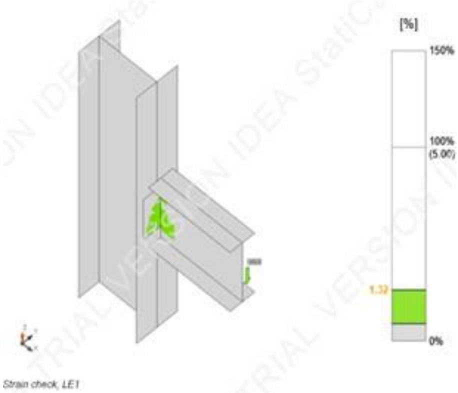
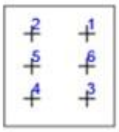


Fig. 6 The Joint Checks Completed

Bolts



Name	Loads	$F_{t,Ed}$ [kN]	V [kN]	Ut_t [%]	$F_{b,Rd}$ [kN]	Ut_s [%]	Ut_{ts} [%]	Status
B1	LE1	1.8	30.6	0.9	199.4	22.5	23.2	OK
B2	LE1	1.9	30.6	0.9	199.4	22.6	23.2	OK
B3	LE1	115.7	33.4	56.9	259.2	24.7	65.3	OK
B4	LE1	115.6	33.4	56.9	259.2	24.6	65.3	OK
B5	LE1	35.5	32.7	17.5	167.8	24.1	36.6	OK
B6	LE1	35.5	32.7	17.5	167.8	24.1	36.6	OK

Design data

Name	$F_{t,Rd}$ [kN]	$B_{p,Rd}$ [kN]	$F_{v,Rd}$ [kN]
M24 8.8 - 1	203.3	309.4	135.6

Fig. 7 Verified M24 bolts

A capable force of 212kN was determined. At this shear force value, it is observed that the IPE400 beam's web will begin to fail, which is the same failure mode resulting from the analytical calculation according to SR EN 1993-1-8.

Summary

Name	Value	Status
Analysis	100.0%	OK
Plates	5.5 > 5%	Not OK!
Bolts	86.1 < 100%	OK
Welds	0.0 < 100%	OK
Buckling	Not calculated	

Plates

Name	Thickness [mm]	Loads	σ_{Ed} [MPa]	ε_{Pl} [%]	Status
C-bfl 1	26.0	LE1	177.5	0.0	OK
C-tfl 1	26.0	LE1	3.9	0.0	OK
C-w 1	14.0	LE1	100.0	0.0	OK
B-bfl 1	13.5	LE1	156.9	0.0	OK
B-tfl 1	13.5	LE1	156.9	0.0	OK
B-w 1	8.6	LE1	246.5	5.5	Not OK!
EP1	15.0	LE1	245.3	4.9	OK

Fig. 8 The Unfulfilled Joint Checks

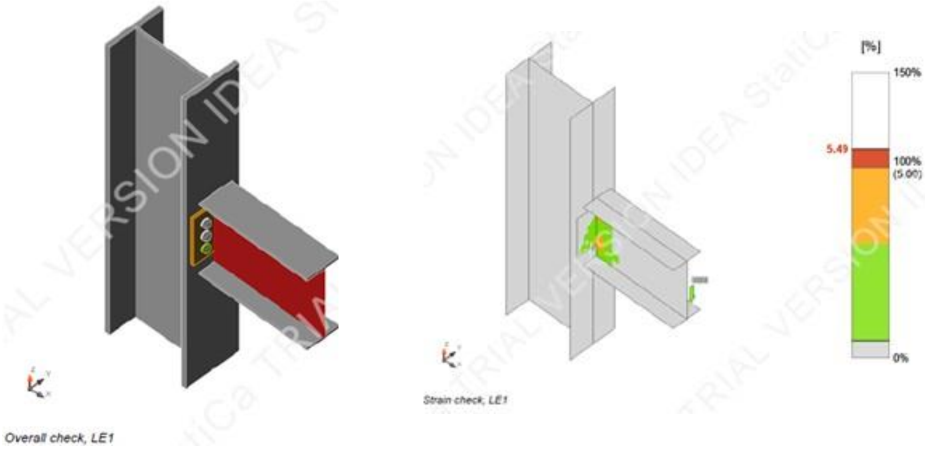


Fig. 9 Global heat transfer through a homogeneous element [9]

4. CONCLUSIONS

For the designed structure, all the strength and stability checks have been carried out and the specific provisions of the P100-1/2013 standard have been taken into account. The overresistances were obtained after the optimization of the sections, obtaining values close to those of Annex F of P-100.

The verifications of the articulated node showed that the differences between the maximum results obtained from the analytical calculation and the results obtained from the program are not significant (273 kN of result from the analytical calculation and 212 kN of result from the IDEA StatiCa program), but it can be concluded that, whenever possible, it is recommended to check with both methods and to take into account the minimum effort capable result. Both the effort that can be achieved through the analytical calculation and that obtained with the help of the IDEA StatiCa program are lower than the calculation effort of the ETABS program of 192.03 kN, which shows us that the joint has the necessary strength.

5. REFERENCES

[1] F. Țepes-Onea, *Metal Constructions*, MatrixRom Publishing House, Bucharest, 2023

[2] P100-1/2013 – *Seismic design code – Design provisions for buildings*

[3] *Calculation and Design of Steel Structural Joints in accordance with SR-EN1992-1*

[4] *Recommendations, observations and examples of application (Contract no. 426/08.12.2009)*, Timişoara, August 2010

[5] *Verification of the stability of steel elements according to SR-EN 1992-1-1. Recommendations, observations and examples of application (Contract no. 424/08.12.2009)*, Timişoara, August 2010

[6] *Global structural calculation of metallic structures. Recommendations, comments and examples of application (Contract no. 425/08.12.2009)*, Timisoara, August 2010

[7] *SR EN 1993-1-8: 2006 Eurocode 3: Design of steel structures - Part 1.8: Design of joints*. Romania, 2006

[8] http://www.reflex-ing.ro/IDEAStatiCa/idea-statica-steel-connection-ro/